4.3 NOTATION

Revise the following definitions:

d_e = distance from the centerline of the exterior web of exterior beam to the interior edge of curb or traffic barrier (ft.)

4.4 ACCEPTABLE METHODS OF STRUCTURAL ANAYSIS

Delete the 3rd paragraph as follows:

~~The name, version, and release date of software used should be indicated in the contract documents.~~

4.6.2.2 Beam-Slab Bridges

4.6.2.2.1 Application

Revise 1st and 6th paragraph as follows:

The provisions of this Article may be applied to superstructures modeled as a single spine beam, straight girder bridges and horizontally curved concrete bridges, as well as horizontally curved steel girder bridges complying with the provisions of Article 4.6.1.2.4. The provisions of this Article may also be used to determine starting point for some methods of analysis to determine force effects in curved girders of any degree of curvature in plan.

Bridges not meeting the requirements of this article shall be analyzed as specified in Article 4.6.3, or as directed by the Owner.

Add a sentence to the 9th paragraph as follows:

Cast-in-place multicell concrete box girder bridge types may be designed as whole-width structures. Such cross-sections shall be designed for the live load distribution factors in Articles 4.6.2.2.2 and 4.6.2.2.3 for interior girders, multiplied by the number of girders, i.e., webs. The live load distribution factors for moment shall be applied to maximum moments and coincident moments. The live load distribution factors for shear shall be applied to maximum shears and coincident shears.

C4.6.2.2.1

Revise the 8th paragraph as follows:

Whole-width design is appropriate for torsionally-stiff cross-sections where load-sharing between girders is extremely high and torsional loads are hard to estimate. Prestressing force should be evenly distributed between girders. Cell width-to-height ratios should be approximately 2:1. The distribution factors for exterior girder moment are not used because the difference in total number of design lanes doesn't change appreciably when doing so. The two or-more-lanes loaded distribution factors for exterior girder shear are not used because, at most, a 4% increase would occur due to the range-of-applicability of d_e. The one-design-lane-loaded distribution factors for exterior girder shear are not used because lever rule is inappropriate for use in multi-cell boxes.

4.6.2.2.2e Skewed Bridges

Delete the 1st paragraph as follows:

~~When the line supports are skewed and the difference between skew angles of two adjacent lines of supports does not exceed 10°, the bending moment in the beams may be reduced in accordance with Table 1.~~

C4.6.2.2.2e

Revise the 1st paragraph as follows:

Accepted reduction factors are not currently available for cases not covered in Table 1. <u>Caltrans presently does not take advantage of the reduction in load distribution factors for moment in longitudinal beams on skewed supports.</u>

4.6.2.2.3c Skewed Bridges

Revise as follows:

Shear in the exterior and first interior beams on ~~at~~ the obtuse side ~~corner~~ of the bridge shall be adjusted when the line of support is skewed. The value of the correction factor shall be obtained from Table 1. It is applied between the point of support and midspan to the lane fraction specified in Table 4.6.2.2.3a-1 for interior beams and in Table 4.6.2.2.3b-1 for exterior beams.

~~In determining the end shear in multibeam bridges, the skew correction at the obtuse corner shall be applied to all the beams.~~

When shear-moment interaction is taken into consideration, the factors apply to shear only, and not to associated moments in longitudinal strain and longitudinal steel calculations. For curved bridges having large skews (> 45°), the designer shall consider a more refined analysis that also considers torsion.

C4.6.2.2.3c

Add Commentary as follows:

The factors in Table 1 may decrease linearly to a value of 1.0 at midspan, regardless of end condition.

The factors in Table 1 were developed by comparing grillage analyses of simple-span straight bridges with equivalent skewed structures, for application to a two-dimensional frame design. Shear and moment interaction was not considered. The adjusted force effects may be overly conservative when skewed bents, abutments, and superstructure are modeled in intersecting planes with corresponding support conditions. A three-dimensional finite-element shell/plate or grillage model in compliance with Articles 4.5 and 4.6.3 may be used.

For curved bridges having large skews (> 45°), the designer should consider a more refined analysis that also considers torsion.

Revise Table 4.6.2.2.3c-1 as follows:

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Correction Factor	Range of Applicability
Cast-in-place Concrete Multicell Box	d	$\cancel{1.0+\left(0.25+\frac{12.0L}{70d}\right)\tan\theta}$ $1.0+\frac{\theta}{50}$ for exterior girder $1.0+\frac{\theta}{300}$ for first interior girder	$0 < \theta \leq 60^{o}$ $6.0 < S \leq 13.0$ $20 \leq L \leq 240$ $35 \leq d \leq 110$ $N_c \geq 3$

4.6.2.2.5 Special Loads with other Traffic

Revise the 1st paragraph as follows:

Except as specified herein, the provisions of this article may be applied where the approximate methods of analysis for the analysis of beam-slab bridges specified in Article 4.6.2.2 and slab-type bridges specified in Article 4.6.2.3 are used. The provisions of this article shall not be applied where either:

- the lever rule has been specified for both single lane and multiple lane loadings, or
- the special requirement for exterior girders of beam-slab bridge cross-sections with diaphragms specified in Article 4.6.2.2.2d has been utilized for simplified analysis.
- Two identical permit vehicles in separate lanes are used, as specified in CA amendment to Article 3.4.1.

4.6.2.2.6 Permanent Loads Distribution

Add new articles:

4.6.2.2.6a- Structural Element Self-Weight

Shears and moment due to the structural section self-weight shall be distributed to individual girders by tributary area methods. For CIP concrete multi-cell box girder bridges, the self-weight shears in the exterior and first interior beams on the obtuse side of the bridge shall be adjusted when the line of support is skewed similar to live load shears in Article 4.6.2.2.3c.

4.6.2.2.6b- Non-Structural Element Loads

Non-structural loads include appurtenances, utilities, wearing surface, futures overlays, and earth cover. Curbs and wearing surfaces, if placed after the slab has been cured, may be distributed equally to all roadway stringers or beams. Barrier loads are less significant and may be equally distributed to all girders. For CIP concrete multi-cell box girder bridges, the non-structural element shears in the exterior and first interior beams on the obtuse side of the bridge shall be adjusted when the line of support is skewed similar to live load shears in Article 4.6.2.2.3c.

4.6.2.5 Effective Length Factor, *K*

Revise as follows:

Physical ~~column~~ lengths of compression members shall be multiplied by an effective length factor, *K*, to compensate for rotational and translational boundary conditions other than pinned ends.

In the absence of a more refined analysis, where lateral stability is provided by diagonal bracing or other suitable means, the effective length factor in the braced plane, *K*, for the compression members shall be taken as unity, unless structural analysis shows a smaller value may be used. In the absence of a more refined analysis, the effective length factor in the braced plane for steel in ~~triangulated trusses,~~ trusses~~, and frames~~ may be taken as:

- For compression chords: $K = 1.0$
- For bolted or welded end connections at both ends: $K =$ ~~0.750~~ 0.850
- ~~For pinned connections at both ends: *K* = 0.875~~
- For single angles, regardless of end connection: $K = 1.0$

Vierendeel trusses shall be treated as unbraced frames.

C4.6.2.5

Revise the 1st and 2nd paragraphs as follows:

Equations for axial ~~the compressive~~ resistance of columns and moment magnification factors for beam-columns include a factor, *K*, which is used to modify the length according to the restraint at the ends of the column against rotation and translation.

K is the ratio of the effective length of an idealized pin-end column to the actual length of a column with various other end conditions. *KL* represents the length between inflection points of a buckled column influenced by the restraint against rotation and translation of column ends. a factor that when multiplied by the actual length of the end-restrained compression member, gives the length of an equivalent pin-ended compression member whose buckling load is the same as that of the end-restrained member. The Structural Stability Research Council (SSRC) Guide (Galambos 1988) recommends $K = 1.0$ for compression chords on the basis that no restraint would be supplied at the joints if all chord members reach maximum stress under the same loading conditions. It also recommends $K = 0.85$ for web members of trusses supporting moving loads. The position of live load that produces maximum stress in the member being designed also results in less than maximum stress in members framing into it, so that rotational restraint is developed. Theoretical values of *K*, as provided by the Structural Stability Research Council, are given in Table C1 for some idealized column end conditions.

4.6.2.6 Effective Flange Width

4.6.2.6.1 General

Revise as follows:

In the absence of a more refined analysis and/or unless otherwise specified, limits of the width of a concrete slab, taken as effective in composite action for determining resistance for all limit states, shall be as specified herein. The calculation of deflections should be based on the full flange width. For the calculation of live load deflections, where required, the provisions of Article 2.5.2.6.2 shall apply.

The effective span length used in calculating effective flange width may be taken as the actual span for simply supported spans and the distance between points of permanent load inflection for continuous spans, as appropriate for either positive or negative moments.

The effective flange width may be taken as:

If $S/L \leq 0.32$, then:

$$b_{eff} = b \qquad (4.6.2.6.1\text{-}1)$$

Otherwise:

$$b_{eff} = \left[1.24 - 0.74\left(\frac{S}{L}\right)\right]b \geq b_{min} \qquad (4.6.2.6.1\text{-}2)$$

where

b = full flange width (ft)

b_{eff} = effective flange width (ft)

b_{min} = minimum effective flange width (ft)

L = span length (ft)

S = girder spacing (ft)

Equations 1 and 2 shall be used within the limit of skew angle $\theta \leq 60^o$. For $\theta > 60^o$, unless a more refined analysis is performed, the effective flange width may be taken as b_{min} and shall not exceed the girder spacing.

C4.6.2.6.1

Revise as follows:

Longitudinal stresses in the flanges are spread across the flange and the composite deck slab by in-plane shear stresses. Therefore, the longitudinal stresses are not uniform. The effective flange width is ~~a reduced~~ the width over which the longitudinal stresses are assumed to be uniformly distributed and yet result in the same force as the nonuniform stress distribution would if integrated over the whole width.

The effective flange width provisions are based on state-of-the-art research by Chen, et al. (*2005*), Nassif et al. (*2005*), and Caltrans revisions. The concrete deck slabs shall be designed in accordance with Article 9.7.

The girder spacing and the full flange width are shown in Figure C1. For interior beams, the girder spacing, *S*, and the full flange width, *b*, shall be taken as the average spacing of adjacent beams. For exterior beams, the girder spacing, *S*, and the full flange width, *b*, shall be taken as the overhang width plus one-half of the adjacent interior beam spacing, and shall be limited to the adjacent interior beam spacing.

$$b = S = \frac{S_1}{2} + \frac{S_2}{2} \qquad b = S = \frac{S_1}{2} + S_o \leq S_1$$

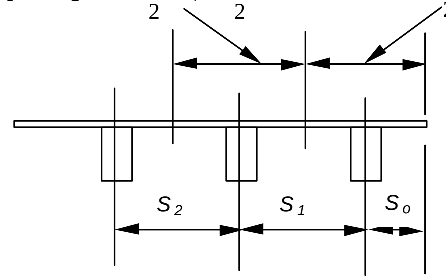

Figure C4.6.2.6.1-1 Girder Spacing and Full Flange Width.

The full flange width is proposed within the limits of the parametric study ($S \leq 16$ ft, $L \leq 200$ ft, $\theta \leq 60^o$) by Chen et al. (*2005*) based on an extensive and systematic investigation of bridge finite element models. The full flange width is also proposed within the limit of $S/L \leq 0.25$ by Nassif et al. (*2005*). For $S/L > 0.25$, Nassif et al. (*2005*) recommends that:

$$\frac{b_{eff}}{b} = 1.0 - 0.5\left(\frac{S}{L}\right) \qquad (C4.6.2.6.1\text{-}1)$$

Figure C2 shows a graphic illustration of Equation 1 which is a good combination of the effective flange width criteria proposed by Chen et al. (*2005*) and Nassif et al. (*2005*). For $S/L \le 0.32$, the exact parametric study limit adopted by Chen et al. (*2005*), Equation 1 gives the full flange width. For $S/L = 1$, Equation 1 provides one-half of the full flange width which is as same as Equation C1.

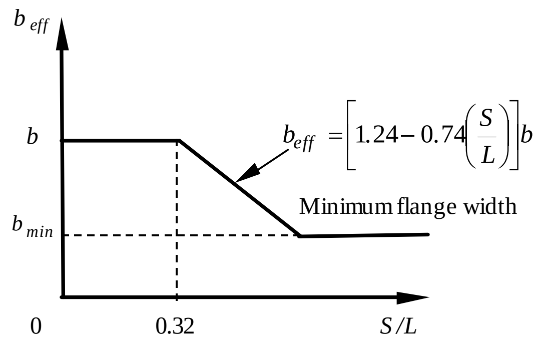

Figure C4.6.2.6.1-2 Effective Flange Width

For interior beams, the minimum effective flange width, b_{min} ~~effective flange width~~ may be taken as the least of:

- One-quarter of the effective span length;
- 12.0 times the average depth of the slab, plus the greater of web thickness or one-half the width of the top flange of the girder. ~~; or~~
- ~~The average spacing of adjacent beams.~~

For exterior beams, the minimum effective flange width, b_{min} ~~effective flange width~~ may be taken as one-half the effective width of the adjacent interior beam, plus the least of:

- One-eighth of the effective span length;
- 6.0 times the average depth of the slab, plus the greater of one-half the web thickness or one-quarter of the width of the top flange of the basic girder.~~; or~~
- ~~The width of the overhang.~~

~~In calculating the effective flange width for closed steel and precast concrete boxes, the distance between the outside of webs at their tops will be used in lieu of the web thickness, and the spacing will be taken as the spacing between the centerlines of boxes.~~

~~For open boxes, the effective flange width of each web should be determined as though each web was an individual supporting element.~~

~~For filled grid, partially filled grid, and for unfilled grid composite with reinforced concrete slab, the "slab depth" used should be the full depth of grid and concrete slab, minus a sacrificial depth for grinding, grooving and wear (typically 0.5 in.).~~

When $S/L > 0.32$, the effective flange width calculated by Equation 1 is less than the full flange width as shown in Figure C2. When $S/L > 1.68$, especially for commonly used bent cap beams, the effective flange width calculated by Equation 1 is less than zero. Since the effective flange width can not logically be less than zero, based on the past successful practice the meaningful lower limit, the minimum effective flange width, b_{min}, is added in Equation 1. The minimum effective flange width, b_{min} should be checked when $S/L > 0.32$.

For negative moment region only, one possible alternative for determining the effective flange width is provided by Equation C2 is more accurate:

$$\frac{b_{eff}}{b} = 0.948 + 0.003\left(\frac{L}{S}\right) - 0.001\theta \leq 1.0 \quad \text{(C4.6.2.6.1-2)}$$

where

,L = span length (ft), the lesser of the two span lengths if the two span lengths differ

θ = skew angle (°)

By comparing the results using the effective flange width obtained from the finite element analyses and a full slab width, the difference can be as high as 8.5%. By using Equation C2 the difference can be reduced to approximately 5.9% in the worst case investigated by Chen et al. *(2005*).

Both the full physical flange width provision and Equation C2 were formulated based on finite element models that developed slab cracking in the negative moment sections under service loads. Thus, in negative moment regions these provisions should be used assuming the slab to be cracked, i.e., the composite section to consist of the beam section and the longitudinal reinforcement within the effective width of concrete deck.

A more refined analysis should be performed to determine the effective flange width when $\theta > 60°$.

In calculating the effective flange width for closed steel and precast concrete boxes, ~~the distance between the outside of webs at their tops will be used in lieu of the web thickness, and~~ the girder spacing will be taken as the spacing between the centerlines of adjacent boxes.

For open boxes, the effective flange width of each web should be determined as though each web was an individual supporting element.

~~For filled grid, partially filled grid, and for unfilled grid composite with reinforced concrete slab, the "slab depth" used should be the full depth of grid and concrete slab, minus a sacrificial depth for grinding, grooving and wear (typically 0.5 in.).~~

Where a structurally continuous concrete….

For integral bent caps, the effective flange width overhanging each side of the bent cap web shall not exceed six times the least slab thickness, or 1/10 the span length of the bent cap. For cantilevered bent caps, the span length shall be taken as two times the length of the cantilever span.

The provisions for the effective flange width for the integral bent cap are based on past successful practice, specified by Article 8.10.1.4 of the 2002 AASHTO Standard Specifications.

4.6.3 Refined Methods of Analysis

4.6.3.1 General

Revise the 2nd paragraph as follows.

~~A structurally continuous railing, barrier, or median, acting compositely with the supporting components, may be considered to be structurally active at service and fatigue limit states.~~ Railings, barriers, and medians shall not be considered as structurally continuous.

C4.6.3.1

Revise the 2nd paragraph as follows:

This provision reflects the experimentally observed response of bridges. This source of stiffness has traditionally been neglected but exists and may be included, proved that full composite behavior is assured. The appurtenances shall not be considered as a structural member because

- Bar splices and development lengths aren't necessarily adequate
- Possible future replacement with a different type of member.

4.6.3.2 Decks

Revise the 1st paragraph as follows:

Unless otherwise specified, flexural and torsional deformation of the deck shall be considered in the analysis but vertical shear deformation may be neglected. Yield-line analysis shall not be used.

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